Finite element simulation of an embankment on soft clay – Case study

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1. Introduction

Comparing the predicted and field-measured behaviour of embankments constructed on soft clayey ground often provides a good check on the suitability of the constitutive model adopted for the clay soil, as well as the capacities of the numerical procedures used to make predictions of the embankment behaviour, providing of course that appropriate values have been adopted for the model parameters. The latter is a function of the quality of sampling, sample preparation, testing and overall characterisation of the soils at the site. There is a relatively rich literature in this particular field of geotechnical engineering (e.g., [30,22,4,6,9,12,15,20,16,21,18]).

The Modified Cam Clay (MCC) constitutive model [23] is one of the most widely used elastoplastic models for soft clayey soils because of its simplicity and its ability to predict yielding, strain softening as well as failure in soft clayey soils. However, MCC is an isotropic yielding model and it does not consider viscous behaviour such as creep of clayey soils. Elasto-viscoplastic (EVP) models (e.g., [31]) and anisotropic yielding elastoplastic models (e.g., [11]) have been developed to account for these complicating effects. However, there are still differing opinions as to which model provides the best prediction of the field response of clayey soils under embankment loading (e.g. [18,20]). In principle, a more sophisticated soil model should be able to represent better the mechanical behaviour of natural soft clayey soils, but the drawback in adopting them is that more sophisticated models require specification of more soil parameters, and in many cases in engineering practice there are insufficient test data to reliably define those parameters. Further, for some sophisticated soil models, some of the soil parameters can only be calibrated by fitting the model predictions to the test results, rather than being determined directly from those test results. On the other hand, the parameters of relatively simple soil models, such as the MCC model, which is acknowledged for being capable of capturing the most important mechanical features of soft clayey soil, can be easily and reliably defined. It is generally acknowledged that the use of some simple models can result in acceptable predictions of soil behaviour, at least from a practical...
perspective. Whichever the case, obtaining accurate Class-A predictions [17] of the behaviour of embankments on soft clayey deposits still remains a difficult task.

As shown in Fig. 1, in Saga, Japan, a highway around the Ariake Sea has been planned and was under construction at the time of writing. For its entire length this highway is located over deposits of soft Ariake clay. In order to verify the correctness of the assumed design strength and deformation parameters of the soft deposit, as estimated from laboratory tests, a test embankment was built on the natural deposit and its performance was monitored for more than 3 years in terms of settlements, lateral displacements and excess pore water pressures. For this test embankment an internal Class-A prediction was made and documented before construction. After the field-measured data became known, these Class-A predictions were compared with the measurements and there were considerable discrepancies. The same case was then re-analyzed (Class-C prediction) using additional site investigation results and after applying a correction to the measurements of the undrained shear strength ($S_u$) of the subsoils. For all the analyses reported here, the soft Ariake clay was modelled by the MCC model.

In this paper the site conditions, history of the embankment construction and the measured data are reported and compared together with the results of the Class-A and Class-C predictions. The insights gained into predicting the behaviour of an embankment on soft ground are discussed.

2. Soil profile and embankment construction

In total, three (3) test embankments were constructed at the location indicated in Fig. 1, one on natural soft ground and two on the same type of soft ground after it had been improved by the installation of soil-cement columns formed by deep mixing [13]. The test embankment considered in this study is the one constructed on natural ground. As shown in Fig. 2, in and around the test site, six (6) boreholes (BH) were drilled in order to investigate the soil properties required for design of the road embankment. At the locations of BH-1, -3, -5 and -6, undisturbed soil samples were taken using a Japanese thin-wall sampler and laboratory index tests and consolidation and unconfined compression tests were conducted on these samples.

BH-1 to BH-5 were drilled and the corresponding laboratory tests were conducted before the test embankment was constructed, and the resulting data were available at the time the Class-A prediction was made. BH-6 was in the test site and was bored just before the commencement of embankment construction. The test data from this borehole were not available when the Class-A predictions were made. The soil profile and some physical properties obtained from BH-1, 3, 5 and 6 are summarized in Fig. 3. In this figure, $W_p$, $W_l$, and $W_c$ are plastic limit, liquid limit and natural water contents respectively, $\gamma_t$ is the unit weight, and $e_0$ is the initial void ratio. Values of $W_p$ and $W_l$ are only for the samples from BH-6.

It is noted that at this site there exists a surface layer about 1.5 m thick underlain by a thick soft silty clay layer (the Ariake clay) which is about 8.0 m thick. Below it is an organic clayey soil layer about 0.3 m thick, underlain by alternating clayey sand and sandy clay layers. The natural water content of the soft silty clay was generally more than 100% and larger than the corresponding liquid limit. The groundwater level was about 1.0 m below the ground surface.

For the undisturbed samples from BH-6, consolidated undrained triaxial compression tests with excess pore water pressure measurement were also conducted. The effective stress paths in a $p' - q$ plot ($p'$ is effective mean stress and $q$ is deviator stress) of the triaxial test are given in Fig. 4. In the figure, $\sigma_1$ and $\sigma_3$ are effective stresses in the vertical and horizontal directions, respectively. The $e - \log(\sigma'_c)$ curves ($e$ is void ratio and $\sigma'_c$ is vertical consolidation stress) from odometer test results for the samples from BH-6 are given in Appendix A.

For the test embankment constructed on natural ground the critical height was estimated to be about 3.0 m, and so it was decided that an embankment should be constructed with an initial
The embankment was only constructed to a total fill thickness of 2.5 m. After the fill reached this thickness the monitored results indicated that the rate of lateral displacement was increasing quickly and after stopping the placement of fill the lateral movement continued with the rate for about a week. The supervisor in charge of this project decided to cease further filling altogether, because building the embankment to failure was not an option. The base dimensions (length × width) of the embankment were 46.8 m × 21.8 m and its side slopes were 1:1.8 (V:H), which resulted in plan dimensions of the top of the embankment of 37.8 m × 13.8 m. Decomposed granite was used as fill material and the average filling rate was about 0.05 m/day. The average total unit weight of the embankment fill as placed was about 19.0 kN/m$^3$. A cross-section of the embankment and some of the key instrumentation points for measuring settlements, lateral displacements and pore water pressures are shown in Fig. 5. The field-measured data presented in the following sections were all sourced from [1].

### 3. Predictions and comparison with field measurements

#### 3.1. Finite element modelling

Predictions were made using plane strain finite element analyses (FEA). The finite element mesh and the boundary conditions adopted are illustrated in Fig. 6. Due to symmetry, only half the embankment was modelled, and the modelled area had an overall horizontal width of 60.0 m and a vertical thickness of 22.5 m. At the left and the right boundaries the horizontal displacement was fixed but vertical displacement was allowed. At the bottom boundary both the horizontal and vertical displacements were fixed. Both the ground surface and the bottom boundaries (sand layer) were considered completely permeable, i.e., they were drainage boundaries, and both the left and right boundaries were considered to be impermeable. Eight-noded quadrilateral elements with excess pore water pressure degrees of freedom at only the four (4) vertex nodes were used to represent the foundation soil. Eight-noded quadrilateral elements without the excess pore water pressure degrees of freedom were used to represent the embankment. The adequacy of the adopted mesh was checked by also using a much finer mesh, by almost quadrupling the number of elements. By comparison of solutions it was found that the mesh originally adopted is capable of providing accurate and converged predictions of the nodal deformations.

In the numerical analyses, the soft clayey layers were represented by the Modified Cam Clay (MCC) stress–strain model [23], and the sand layers and the embankment fill material were simulated by an elastoplastic stress–strain model which obeys the Mohr–Coulomb failure criterion.

The predictions were conducted using a fully coupled finite element consolidation analysis and the program used for the numerical calculations was CRISP-AIT [7], which is based on the original CRISP program [5]. Large deformations were considered approximately by updating the nodal coordinates at the end of each incremental step. Further, to ensure that the true stress–strain law is followed closely in the numerical model, the Newton–Raphson method with an explicit sub-stepping technique.
that includes error control \cite{26} has been incorporated into the finite element program.

3.2. Class-A predictions and comparisons with measured data

3.2.1. Model parameters

The model parameters adopted for the Class-A prediction were estimated from the laboratory test results using the undisturbed samples obtained from BH-1, -3, and -5. The location of these boreholes is referred to in the following as the ‘Nearby Area’. The parameters values deduced from the laboratory testing are listed in Table 1. In this table the values of Poisson’s ratio, \(\nu\), were assumed to be 0.3. Values of \(K\) were assumed as 0.1. At the time of making the Class-A prediction, there were no test results that allowed an independent estimation of the friction angle (\(\phi^\prime\)).

The values of Young’s modulus (\(E\)) of the sandy layers were estimated from standard penetration test \(N\)-values as \(E = 2500\text{N} / \text{kPa}\) \cite{14}. The values of \(k_0\) were estimated as twice the values deduced from the laboratory incremental loading consolidation tests, and the values of horizontal hydraulic conductivity, \(kh\), were set as 1.5 times the corresponding value of \(k_h\), based on previous experience with this soil \cite{8}. The values of \(k_0\) and \(k_h\) listed in Table 1 are initial values and during consolidation they were allowed to vary with void ratio according to the following equation \cite{29}:

\[
k = k_0 \cdot 10^{(e_0 - e) / k_h}
\]

where \(k_0\) is the initial hydraulic conductivity, \(e_0\) the initial void ratio, \(k\) the current hydraulic conductivity, \(e\), the current void ratio, and \(C_k\) is a constant. Tavenas et al. \cite{28} suggested that generally \(C_k = (0.4–0.5)e_0\) and in this study \(C_k = 0.4e_0\) was assumed.

The overconsolidation ratio (OCR) of the soft soil is a key parameter controlling the amount of deformation of the deposit under the embankment loading. It is linked with the size of yield locus (\(p_y\)) and therefore the undrained shear strength (\(S_u\)), as predicted by the MCC model. According to this model and for undrained triaxial compression stress paths, \(S_u\) value predicted by MCC can be expressed by the following equation:

\[
S_u = \frac{p_y}{2\gamma_0 M (\frac{M^2 + \eta^2}{M^2 + \eta^2} + \frac{\Delta}{\gamma_0^2})^{\frac{1}{2}}}
\]

where \(\Delta = 1 - \kappa / \lambda\), and \(\eta = q / p_y\). Determining the value of the preconsolidation stress (\(p_c\)) and therefore the value of OCR from the incremental loading laboratory consolidation test results may depend on the judgment of the person interpreting the test results. To avoid completely subjective judgment, the approach adopted was to adjust the value of \(p_c\) by comparing the measured value of \(S_u\) with the value predicted by Eq. (2). The measured value of \(S_u\) was assumed to be \(S_u = q_0 / 2\), where \(q_0\) is the measured unconfined compression strength of the undisturbed soil sample. In order to predict the value of \(S_u\) from the MCC model, Mayne and Kulhawy’s \cite{19} equation was used to calculate the coefficient of earth pressure at-rest (\(k_0\)) and therefore the initial effective stress in the ground in the horizontal direction. This value of \(k_0\) was computed by assuming an internal friction angle of the subsoil of 30°.

Measured data from BH-1, -3, and -5 (open circles) and the predicted values (dashed lines) are shown in Figs. 7a and 8a for \(S_u\) (labelled ‘Su-A&C1’) and OCR (labelled ‘OCR-A’), respectively. The

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil strata</th>
<th>SPT N</th>
<th>(E) (kPa)</th>
<th>(\nu)</th>
<th>(\kappa)</th>
<th>(\lambda)</th>
<th>(M)</th>
<th>(e_0)</th>
<th>(\gamma_t) (kN/m^3)</th>
<th>(k_r)</th>
<th>(k_h)</th>
</tr>
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<tbody>
<tr>
<td>0.0–1.5</td>
<td>Surface soil</td>
<td>–</td>
<td>–</td>
<td>0.30</td>
<td>0.025</td>
<td>0.25</td>
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<td>1.50</td>
<td>16.0</td>
<td>6.0</td>
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<td>1.5–4.0</td>
<td>Soft silty clay</td>
<td>–</td>
<td>–</td>
<td>0.30</td>
<td>0.055</td>
<td>0.65</td>
<td>1.2</td>
<td>3.14</td>
<td>13.7</td>
<td>5.1</td>
<td>7.7</td>
</tr>
<tr>
<td>4.0–6.0</td>
<td></td>
<td>–</td>
<td>–</td>
<td>0.30</td>
<td>0.059</td>
<td>0.59</td>
<td>1.2</td>
<td>2.89</td>
<td>13.9</td>
<td>5.4</td>
<td>8.1</td>
</tr>
<tr>
<td>6.0–8.0</td>
<td></td>
<td>–</td>
<td>–</td>
<td>0.30</td>
<td>0.060</td>
<td>0.60</td>
<td>1.2</td>
<td>2.67</td>
<td>14.1</td>
<td>5.4</td>
<td>8.1</td>
</tr>
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<td>8.0–10.0</td>
<td></td>
<td>–</td>
<td>–</td>
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<td>0.71</td>
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<td>2.55</td>
<td>14.3</td>
<td>4.6</td>
<td>6.9</td>
</tr>
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<td>10.0–12.0</td>
<td>Sandy clay</td>
<td>–</td>
<td>–</td>
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<td>0.08</td>
<td>1.2</td>
<td>1.10</td>
<td>18.0</td>
<td>17.5</td>
<td>26.3</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>Soil strata</td>
<td>SPT N</td>
<td>(E) (kPa)</td>
<td>(\phi) (°)</td>
<td>(C) (kPa)</td>
<td>(M)</td>
<td>(e_0)</td>
<td>(\gamma_t) (kN/m^3)</td>
<td>(k_r)</td>
<td>(k_h)</td>
<td></td>
</tr>
<tr>
<td>12.0–15.0</td>
<td>Clayey sand</td>
<td>8</td>
<td>20,000</td>
<td>0.25</td>
<td>35</td>
<td>20</td>
<td>–</td>
<td>0.80</td>
<td>18.0</td>
<td>2500</td>
<td>2500</td>
</tr>
<tr>
<td>15.0–20.0</td>
<td>Clayey sand</td>
<td>15</td>
<td>37,500</td>
<td>0.25</td>
<td>35</td>
<td>20</td>
<td>–</td>
<td>0.70</td>
<td>19.0</td>
<td>2500</td>
<td>2500</td>
</tr>
<tr>
<td>Embankment</td>
<td></td>
<td>2</td>
<td>5000</td>
<td>0.30</td>
<td>35</td>
<td>20</td>
<td>–</td>
<td>15.0</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Note: \(E\) = Young’s modulus; \(\nu\) = Poisson’s ratio; \(\lambda\) = slope of consolidation line in \(e\)-ln\(p\) plot (\(e\) is voids ratio and \(p\) is effective mean stress); \(\kappa\) = slope of rebound line in \(e\)-ln\(p\) plot; \(M\) = strength parameter for Cam-clay model, stress ratio at failure, \(q / p_y\) (\(q\) is deviator stress); \(e_0\) = initial void ratio; \(\gamma_t\) = unit weight; \(k_r\) and \(k_h\) = horizontal and vertical hydraulic conductivity, \(c\) = cohesion, and \(\phi^\prime\) = friction angle of soil.
predicted values of $S_u$ are slightly lower than the average measured values, but the predicted values of OCR form almost an upper bound on the measured data. Initially these discrepancies were judged to be due to unavoidable sample disturbance, i.e., the measured OCR values may have been less than the actual field values. The values of “$S_u$-A&C1” and “OCR-A” shown in Figs. 7a and 8a were adopted in the Class-A prediction of the embankment response.

3.2.2. Comparison of predictions and field measurements

A comparison of the settlement–time curves is given in Fig. 9a–c for settlement gauges $S_0$, $S_1$ and $S_2$, respectively (see Fig. 5 for the locations). There were some problems at settlement gauge $S_2$ and the data were considered unreliable and there were only very small settlements measured at the location of $S_4$ (about 18 mm), so that comparisons for these two locations have been omitted. It can be seen that as for the settlement at the ground surface ($S_0$), the Class-A prediction is at most about 50% of the measured data, i.e., a poor Class-A prediction has been obtained for these settlements.

The simulated settlement at a depth of 9.5 m (Fig. 9c) reduced slightly during the consolidation process. Although it is not very obvious, the measured data also show the same tendency. The reason is probably related to the likelihood that the plastic strain was concentrated largely in the very soft layer, and squeezing the soil sideways meant that there might be slight stress decrease deeper down. It is worth mentioning that from the settlement vs. log time plot it is assumed that primary consolidation finished at about 3 years after the beginning of the construction.

The lateral displacement profiles at the end of the embankment construction and 2 years from the beginning of the construction are compared in Fig. 10a and b, respectively. Although there are discrepancies, compared with the settlement predictions it could be concluded that the Class-A prediction has provided reasonable estimates of the lateral displacements.

Comparisons of the measured and predicted excess pore water pressures ($u$) at the locations $P_1$ to $P_4$ (see Fig. 5 for the locations) are shown in Fig. 11a–d, respectively. Very small values of predicted and measured excess pore water were relevant at location $P_5$ and so a comparison is not presented. It is clear that the Class-A prediction method yielded good estimates of excess pore water pressure ($u$) at locations $P_1$ and $P_2$, but under and over-predicted values of $u$ at locations $P_3$ and $P_4$, respectively. Generally, the Class-A prediction method resulted in acceptable simulations of these excess pore water pressures.
sons of the adopted and measured values of assumed for the model input parameters were checked. Comparison of settlement–time curves. 

Fig. 9. Comparison of settlement–time curves.

3.3. Class-C predictions and comparisons with measured data

3.3.1. Analysis with soil parameters from BH-6

After the results of tests on soil samples recovered from borehole BH-6 (whose location will be referred to as the 'Test Site') became known, the correctness and appropriateness of the values assumed for the model input parameters were checked. Comparisons of the adopted and measured values of $S_0$ and OCR are given in Figs. 7b and 8b, respectively. Although the data are scattered, the adopted values could be considered to be reasonable. However, for the values of $\lambda$ there are considerable differences, as shown in Fig. 12. For soil layers from the ground surface to about 9 m depth, the values of $\lambda$ from BH-6 are generally much larger than those obtained from samples of the Nearby Area. The corresponding initial voids ratio, unit weight and hydraulic conductivity are also different. It is notable that this site was tidal land about 300 years ago [25]. Therefore, the very soft, more compressible spots may reflect the locations of the channels of old creeks or rivers.

Based on the results of BH-6, values of the model parameters, $\lambda$, $K$, $e_0$, $\gamma'$, $k_0$ and $k_r$ were redefined for the soil layers from 1.5 m to 12.0 m, as listed in Table 2. Other modifications are as follows:

1. Soil model: Based on the site investigation results of the Nearby Area, the soil layer from 10.0 m to 12.0 m depth was classified as sandy clay, but the results from the Test Site indicate it is a clayey sand layer with a depth range of 9.8–12.0 m. For this layer, the constitutive model was changed from MCC to an elastoplastic stress–strain model that obeys the Mohr-Coulomb failure criterion and the depths were changed accordingly.

2. $M$ values: From the triaxial compression test results obtained for four different depths at the Test Site (Fig. 4), the slope of a straight line $[\sin(\phi')]$, drawn from the origin to the peak point of each individual effective stress path, reduces as the initial consolidation stress increases. Because MCC is a no-cohesion model, an envelope of best fit passing through the origin was determined by considering the representative stress levels in the deposit under the embankment loading. The redefined values of $M$ (corresponding to a $\phi'$ value of about 39.2° under triaxial compression) are listed in Table 2.

The sizes of the yield loci were subsequently redefined in order to predict the target values of $S_0$, as determined from the unconfined compression tests. However, the values of $K_0$ used to calculate the initial horizontal effective stresses in the deposit were not changed, even though Mayne and Kulhawy’s [19] equation would suggest that a higher internal friction angle ($\phi'$) should result in a slightly smaller value of $K_0$. Furthermore, it is noted that the results of constant rate of strain consolidation tests conducted in the laboratory using undisturbed Ariake clay samples, cut vertically and horizontally with respect to the in situ condition, indicate that the yield stress in the horizontal direction is about 0.5–1.0 (average of about 0.7) times the value in the vertical direction [10]. Thus, for simplicity, it was assumed that values of $K_0$ used to define the initial horizontal effective stress should be left at approximately 0.5–0.6 in the Class-C prediction.

3. Poisson’s ratio: Although the Class-A prediction resulted in a good prediction of the lateral displacement profile, it was considered that the lateral displacements might now be over-predicted if the larger values of the compression parameters, as listed in Table 2, were adopted in the revised analysis. In the Class-C prediction, a value of Poisson’s ratio of 0.15 was adopted for all soil layers (except the embankment fill material).

4. Size of the initial yield loci: Changing the value of $M$ will obviously change the shape of the yield function. As illustrated in Fig. 13, in order to predict the same target value of $S_0$ corresponding to a given initial stress state, the size of the yield locus $[p_y^i]$ for $M = 1.6$ is smaller than that corresponding to $M = 1.2$. Also, with the assumed initial stress state and for undrained triaxial compression, the effective stress path for the case where $M = 1.2$ approaches the critical state line (CSL) from the “dry” side, while for $M = 1.6$ it approaches CSL from the “wet” side. Keeping the values labelled “$S_0$ & C1” in Fig. 7 unchanged, the predicted OCR values were changed to those labelled “OCR-C1” in Fig. 8.

An analysis using the parameters in Table 2 was subsequently conducted. The results of this analysis are given in Figs. 9–11, where they are labelled as Class-C1 predictions. It can be seen that there is considerable improvement in the settlement predictions, but generally these predictions are still smaller than the measured data (Fig. 9). Fig. 11a and b reveal an interesting point about the...
variation of $u$ at location $P_1$ (1.4 m from the ground surface) and $P_2$ (5.0 m from the ground surface). The Class-A prediction showed a slower dissipation rate, but in contrast the Class-C1 prediction indicated much faster dissipation. The reasons for this difference are considered to be: (1) the initial values of the hydraulic conductivities for the layers at depths between 1.5 m and 12.0 m shown in Table 2 are larger than the corresponding values listed in Table 1; and (2) the value of $M$ was increased from 1.2 to 1.6. Due to these changes, the predicted effective stress paths of the soils under the embankment were altered. Fig. 14 shows an example comparison of the effective stress paths for a soil element near point $P_2$.

### 3.3.2. Analysis with modified values of $S_u$ and OCR (Class-C2)

It is well known that for clayey soils there is an effect of strain rate on the undrained shear strength, $S_u$ (e.g., [2,3]). In addition, the soil at the site was micro-structured and has exhibited some strain-softening behaviour, as indicated by the effective stress paths in Fig. 4. In the Class-A and Class-C1 analyses, the unconfined compression strengths, $q_u$, obtained from the undisturbed soil samples loaded at an axial strain rate of 1%/min, were used directly to calibrate the yield stress. Therefore, a possibility exists that the deduced values of $S_u$, and hence the corresponding values of OCR, might not be fully representative of field conditions and it is possi-
ble that they have been over-estimated. For this reason further analyses with modified values of \( S_u \) and OCR were conducted. This further modification was made by multiplying the values of \( S_u \) ("\( S_u - A&C1 \)" in Fig. 7) by a factor \( \alpha \), which was assigned values of 0.75, 0.8 and 0.85. The corresponding values of OCR were also re-estimated. By comparing the various predictions with the field measured values, it was found that a value of \( \alpha = 0.85 \) results in the best predictions, and this particular analysis is designated here as Class-C2. The values of \( S_u \) and OCR for the Class-C2 analysis are plotted in Figs. 7 and 8, where they are labelled "\( S_u - C2 \)" and "OCR-C2", respectively. The factor \( \alpha = 0.85 \) was applied to the measured values of \( S_u \) and the results are designated as "Corrected (\( S_{uC} \))" in Fig. 7.

The results of the Class-C2 analysis are compared with the measured and other simulated results in Figs. 9–11 for settlements, lateral displacements and excess pore water pressures, respectively. From Fig. 9, it can be seen that the Class-C2 analysis resulted in a good simulation of the settlement–time curves, although it still slightly under-estimated the settlement magnitudes. The results in Fig. 10 indicate that the Class-C2 prediction slightly over-estimated the lateral displacements, but overall the predictions are very good. As for the excess pore water pressures, generally the results from the Class-C2 predictions are excellent, except for the values at location P1 where the simulated dissipation rate was faster than the measured rate (Fig. 11a).

### 3.4. Significance of the factor \( \alpha \)

The main reasons for operative field values of \( S_u \) being less than the laboratory measured values are: (a) strain rate effects, and (b) de-structuring or strain softening of the micro-structured (or sensitive) soil. Therefore \( \alpha \) can be regarded as a correction factor for the effects of strain rate and/or strain-softening.

### Table 2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil strata</th>
<th>SPT N</th>
<th>( E ) (kPa)</th>
<th>( \nu )</th>
<th>( \kappa )</th>
<th>( \lambda )</th>
<th>( M )</th>
<th>( e_0 )</th>
<th>( \gamma_c ) (kN/m(^3))</th>
<th>( k_s )</th>
<th>( k_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(10(^{-4}) m/day)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0–1.5</td>
<td>Surface soil</td>
<td>– –</td>
<td>0.15</td>
<td>0.025</td>
<td>0.25</td>
<td>1.6</td>
<td>1.50</td>
<td>16.0</td>
<td>6.0</td>
<td>9.1</td>
<td></td>
</tr>
<tr>
<td>1.5–4.0</td>
<td>Soft silty clay</td>
<td>– –</td>
<td>0.15</td>
<td>0.107</td>
<td>1.07</td>
<td>1.6</td>
<td>3.71</td>
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<tr>
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<td>– –</td>
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<td>0.084</td>
<td>0.84</td>
<td>1.6</td>
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<td>14.1</td>
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<td>0.066</td>
<td>0.66</td>
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<td>7500</td>
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<td>35</td>
<td>20</td>
<td>–</td>
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<td>20,000</td>
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<td>35</td>
<td>20</td>
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<td>Clayey sand</td>
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<td>–</td>
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As for the strain rate effect, Bjerrum [2] proposed a factor (\( \mu \)) to correct the field vane shear strength (\( S_u \)) used in slope stability analysis. He correlated this factor with the plasticity index (PI) of the soil. Tanaka [27] reported a comparison of field vane shear strength (\( S_u \)) with values determined from laboratory unconfined compression test (\( q_u/2 \)) for seven different clayey soils in Japan and indicated that for values of PI of about 40–80, the average ratio \( S_u/(q_u/2) \) is about 1.0. For the soft Ariake clay samples from BH-6, the PI values were 52–76 (in the Nearby Area the PI values were 32–70). With a PI of about 50, the corresponding Bjerrum correction factor (\( \mu \)) will be about 0.8, which is slightly smaller but comparable with the back-estimated value of \( \alpha = 0.85 \). Using a value of \( \alpha = 0.8 \), the predictions are good, but not quite as good as those corresponding to \( \alpha = 0.85 \).
When Bjerrum's correction factor, $\mu$, was first proposed, the effect of de-structuring or strain-softening of the soils considered was not explicitly mentioned. However, since the values of $\mu$ were obtained by comparing the field vane shear strengths with the back-estimated field mobilised strengths, and because it is generally recognised that all natural clayey soils are micro-structured (e.g., [24]), it is highly likely that both the effects of strain rate and strain-softening are included implicitly in the values of $\mu$ proposed by Bjerrum. Although further research is needed to establish a more rational way to determine the strength reduction factor, $\alpha$, at present it is considered that Bjerrum's strain rate correction factor can be used as a reasonable first approximation of the correction factor for the values of $S_u$.

The use of the Bjerrum correction has meant that the OCR values adopted for the Class-C2 analyses are lower than the values determined from laboratory consolidation test results. It is noted that consolidation test results only provide a single point on the yield surface, and the initial yield function (or yield surface) has to be extrapolated from this point by applying the appropriate yield surface, and the initial yield function (or yield surface) has to be extrapolated from this point by applying the appropriate yield surface.

4. Discussion

The case study presented above has revealed at least three important issues regarding prediction of the performance of earth structures on soft soils.

1. Adequate soil investigation: Of course detailed and appropriate site investigation, soil testing and interpretation of the test results are important in all geotechnical designs. This case study has further emphasised this point. For any natural deposit there may be localised variation of the soil properties, and there is additional risk if the design parameters are estimated using only information obtained from nearby areas rather than the specific site in question.

2. Stress parameters of clayey soils: For most practical cases, the effective stress strength parameters, e.g., the cohesion ($c'$) and internal friction angle ($\phi'$) of a clayey soil, are usually interpreted based on the Mohr-Coulomb failure criterion, and normally finite values will be obtained for both $c'$ and $\phi'$. In cases where a no-cohesion soil model, such as MCC, is used to predict the soil response, there is a possibility that the strength parameter $M$ may be under-estimated if the value of $\phi'$ interpreted from the Mohr-Coulomb criterion is adopted directly. This case study has demonstrated the importance of determining the value of $M$ from laboratory tests following the appropriate effective stress path.

3. Performance of the adopted constitutive model: To predict the mechanical behaviour of a clayey deposit under embankment loading using the finite element approach, it is essential to check whether the constitutive model, as well as the adopted values of the model parameters, are able represent well the strength and deformation characteristics of the subsoil. However, there is no well-established procedure or method to conduct this kind of check. The results of this study indicate that comparing the simulated profile of undrained shear strength ($S_u$) (under triaxial compression) with measured values of $S_u$ may be a useful way to check the suitability of a particular model and the values selected for the model parameters. In this process, the effect of strain rate and/or strain-softening on the values of $S_u$ has to be properly considered. The logic behind this approach is that provided parameters like the compression index and the coefficient of consolidation are chosen correctly, the magnitudes of the predicted soil deformations are strongly influenced by the yield stress (i.e., the size of the yield locus), and for most constitutive models for clay, the predicted value of $S_u$ is closely related to the adopted value of the initial yield stress.

It is proposed that a strength correction factor ($\mu$) should be applied to laboratory and field measures of $S_u$. Although further research is needed to determine $\alpha$ values in a fundamental way, the results of this study show that Bjerrum's [2] empirical strain rate correction factor ($\mu$) can be used as a first approximation of the value of $\alpha$.

Although the MCC model adopted in this study does not consider the anisotropic and time dependent behaviour possessed of most natural clayey deposits, it has still resulted in quite good simulation (Class-C2) of the field measured response. It is considered that the corrected $S_u$ value most likely represents to sufficient accuracy the average field-mobilised value of $S_u$ under an embankment load, and using this value of $S_u$ with an isotropic soil model captures the most important features of the mechanical behaviour of soft clayey soil, and generally results in acceptable predictions of that behaviour. However, if the available site investigation data are sufficient to reliably define the necessary parameters for more sophisticated soil models, such as those capturing anisotropy, in principle use of these more sophisticated models should result in improved predictions of soil behaviour.

5. Conclusions

Field measurements and numerical predictions of the behaviour of a test embankment constructed on a deposit of soft Ariake clay in Saga, Japan, have been presented. The numerical predictions were conducted by finite element analysis (FEA) in which the mechanical behaviour of soft clayey soil layers was represented by the Modified Cam Clay model. These involved two types of prediction: Class-A, conducted before; and Class-C, conducted after the embankment was constructed. Comparisons were made between the various predictions and the field measurements in terms of settlements, lateral displacements and excess pore water pressures in the subsoil.

The results of these comparisons indicate that the Class-A prediction resulted in a poor simulation of the field performance, mainly due to over-estimation of the yield stresses (i.e., the sizes of the initial yield loci) of the subsoils and under-estimation of compressibility, hydraulic conductivity and the slope ($M$) of the critical state line. The following observations are suggested by the results of this study.

(a) Detailed on-site soil investigation, correct testing and appropriate interpretation of the test results are essential issues for predicting the behaviour of an earth structure on a soft clayey deposit.

(b) When using a soil model developed in the framework of Critical State Soil Mechanics, the value of parameter $M$ should be determined directly from test result with an appropriate effective stress path.

(c) Calibrating the yield stress by comparing the simulated profile of undrained shear strength ($S_u$) under triaxial compression with the measured data provides an efficient check on
the suitability of the constitutive model, as well as the adopted values of the model parameters, provided that a correction for the effect of strain rate or strain-softening on $S_0$ value is properly considered. The results also indicate that Bjerrum's strain rate correction factor can be adopted as a first approximation of the correction factor applied to field or laboratory measured values of $S_0$.

Appendix A

Oedometer test results of undisturbed samples from BH-6 are presented in Fig. A1.

References


